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The 1989 Edition of the Canadian Cold Formed Steel Design Standard

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THE 1989 EDITION OF THE CANADIAN COLD FORMED STEEL DESIGN STANDARD

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SUMMARY

This paper presents excerpts from the latest edition of the Canadian Standard for the design of cold formed steel structural members (CAN/CSA-S136-M89). Considerable technical changes, reflecting the latest research developments, have been incorporated into this edition of the Standard. These changes are based on an increased understanding of the behaviour of cold formed steel structures, members, and elements and of cold formed steel as a structural material. Some of the more significant technical changes of the Standard are discussed in greater detail. Comparisons with the Standard's predecessor (CAN3-S136-M84), and whenever appropriate, with the 1986 AISI Specifications are also made.

INTRODUCTION

The 1989 edition of S136, CAN/CSA-S136-M89 [4] is based entirely on limit states design principles (LSD) for use with SI (metric) units, however, the designer has the option to use any other consistent system of measurement units. The resistance factors specified in the Standard have been correlated with the load factors as specified in the National Building Code of Canada [12]. For other cases, load factors must be established such that, in conjunction with the resistance factors used in the new Standard, the required level of reliability is maintained.

The major change incorporated into the 1989 edition of the Standard [4] is a total "Unified" effective width approach for the design of compressive elements subject to local buckling. The 1984 edition of S136, CAN3-S136-M84 [3], already contained a common effective width approach for both stiffened and unstiffened compressive elements under uniform compressive stress. However, in the case of members in bending, a reduction in stress for possible web buckling had to be used. This is no longer the case in the 1989 edition of the Standard [4] since the effective width approach also covers cases under stress gradient, hence, making the "Unified" effective width approach universally applicable to all compressive elements. The treatment of compressive elements with edge stiffeners and compressive elements with one intermediate stiffener has also been revised to allow for a partially stiffened case, which was not permissible in the 1984 edition [3].

Other significant changes incorporated into the 1989 edition of the Standard [4] include:

a) Members in bending are now clearly separated into laterally supported and laterally unsupported cases.

b) Changes have been made for laterally unsupported members bending about the centroidal axis perpendicular to the web.

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c) A new Clause has been added for laterally unsupported members bending about the centroidal axis parallel to the web of singly symmetric sections such as channels.

d) A new Clause has been added for cylindrical tubular members in bending.

e) A new Clause has been added for cylindrical tubular compressive members.

f) The designer now has the option of designing wall studs either on the basis of an all steel system being braced by bridging or strapping alone, or by assuming that the sheathing material provides the bracing function for the studs.

g) Changes have been made in the design of arc spot welds and a new Clause has been added for the design of arc seam welds.

Some of these topics will only be highlighted while others will be discussed in detail, such as the "Unified" effective width approach. Also, there is a Commentary [15] on CAN/CSA-S136-M89 to provide the background research upon which the provisions of the Standard are based.

LIMIT STATES DESIGN (LSD)

In limit states design, the resistance of a structural member is checked against the various limit states. For the ultimate limit states resistance, the structural member must retain its load-carrying capacity up to the factored load levels. For serviceability limit states, the performance of the structure must be satisfactory at specified load levels. Specified loads are those prescribed by the National Building Code of Canada [12]. Examples of serviceability requirements include deflection and vibration control. The fundamental safety criterion that must be met is expressed as follows:

Factored Resistance $\geq$ Effect of Factored Loads

The factored resistance is given by the product ϕR, where ϕ is the resistance factor, which is applied to the nominal member resistance, R. The resistance factor is intended to take into account the fact that the resistance of the member may be less than anticipated, due to the variability of material properties, dimensions, and workmanship, and also to take into account the type of failure and uncertainty in the prediction of the resistance. The resistance factor does not, however, cover gross human errors. Human errors cause most structural failures and typically these human errors are "gross" errors. Gross errors are completely unpredictable and are not covered by the overall safety factor inherent in buildings.

The effect of the factored loads is given by:

$$\alpha_D D + \gamma \psi(\alpha_L L + \alpha_Q Q + \alpha_T T)$$

This expression is identical to that given in Part 4 of the National Building Code of Canada [12], as are the values given for the various load factors, α, load combination factors, ψ, and importance factors, γ.

In limit states design, structural reliability is specified in terms of a safety index, β, which is directly related to the structural reliability of the design; hence, increasing β increases the reliability, and decreasing β decreases the reliability. The safety index, β, is also directly related to the load and resistance factors used in the design.

Those responsible for writing a design standard are given the load distribution and load factors and must establish by calibration the resistance factors, ϕ, such that the safety index, β, reaches a certain target value. The technical committee responsible for the 1989 edition of S136 [4] elected to use a target safety index ranging between 3.0 and 4.0, depending on the load action and resistance type (e.g., shear, bending, web crippling, connections). The calibration procedure used to determine the appropriate resistance factors included a computer simulation of the expected load and resistance distributions.

In order to determine the loading for calibration, it was assumed that 80% of cold formed steel is used in panel form (e.g., roof or floor deck, wall panels, etc.) and the remaining 20% for structural sections (purlins, girts, studs, etc.). An effective load factor was arrived at by assuming live to dead load ratios and their relative frequencies of occurrence. Probabilistic studies by Allen [2] show that consistent probabilities of failure are determined for all live to dead load ratios when a live load factor of 1.50 and a dead load factor 1.25 are used.

UNIFIED EFFECTIVE WIDTH CONCEPT

The well-known phenomenon of post-buckling in thin uniformly compressed plate elements is reflected in the effective width concept, used in both Canada and the U.S.A., when computing section properties of stiffened compressive elements. It has been a long standing practice in both countries to compute section properties of stiffened compressive elements on the basis of an effective width concept (i.e., reduced section properties and full limit stress as opposed to a reduced stress on the gross or full section used in the design of unstiffened compressive elements).

The 1984 edition of S136 [3] used an effective width (reduced section properties) approach for both stiffened and unstiffened compressive elements under uniform stress, thus providing the designer with a more consistent design method. This was based on recent research [6,11], showing both analytically and experimentally that it is appropriate to also utilize the post-buckling capacity of unstiffened compressive elements. Thus, in the 1984 edition of S136 [3], the same basic effective width equation applied to both stiffened and unstiffened compressive elements, with only the buckling coefficient of 4.0 (for stiffened elements) and 0.5 (for unstiffened elements) to be specified.

In 1986, AISI [1] introduced a unified effective width approach for both sitffened and unstiffened compressive elements subjected to uniform stress or stress gradient. Winter's basic modified effective width expression [17] is used with the plate buckling coefficient as the variable, depending on the type of compressive element and the stress condition. The 1989 edition of S136 [4] also introduced a unified effective width concept similar to AISI [1], however, S136 elected to present the information in the traditional format of effective width, as presented herein.

Basic Effective Width Expression (Elements in Compression)

When the flat width ratio, $W = w/t$, exceeds W_{lim}, the flat width, w, must be replaced by an effective width. The effective width ratio, $B = b/t$, for strength and serviceability must be determined as follows:

Case I

When $W \le W_{\text{lim}}$ $\qquad B = W$

Case II

When $W > W_{\text{lim}}$ $\qquad B = 0.95\sqrt{kE/f}\left[1 - \frac{0.208}{W}\sqrt{kE/f}\right]; \qquad W_{\text{lim}}=0.644\sqrt{kE/f}$ (1)

Where

$k =$ plate buckling coefficient, to be calculated for each particular case, depending on the type of element (stiffened or unstiffened) and the stress condition.

For strength determination:

$f =$ calculated stress in compressive elements ($\le F_y$) using factored loads and effective section properties.

For serviceability determination:

$f =$ calculated stress in compressive element using specified loads and effective section properties.

Elements Under Uniform Compressive Stress

This includes stiffened and unstiffened compressive elements.

Elements stiffened on each edge by a web or flange

For elements stiffened on each edge by a web or flange, the effective width, $b = Bt$, shall be determined from Eq. (1) using $k = 4$. Figure 1 shows the assumed stress distribution for design of a stiffened flange element and Figure 2 represents a graphic illustration of the effective width expression for a stiffened compressive element under uniform stress. As can be seen, Eq. (1) provides a relatively smooth transition from the fully effective line, $W = B$, to the extreme limiting value of $B = 1.9\sqrt{E/f}$ (von Karman's expression [18]) at large W ratios. This transition, particularly in the region of the knee, reflects the complex interaction between elastic plate buckling, material yield strength, and geometric imperfections for plates with moderate W ratios.

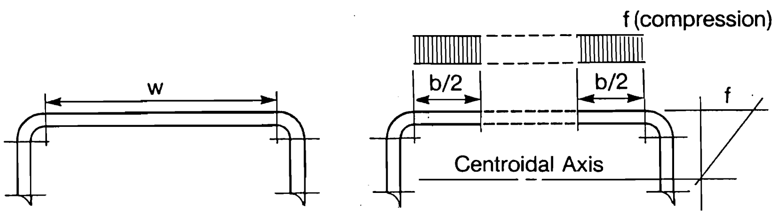

Figure 1 Example of Stiffened Flange Element Under Uniform Compressive Stress

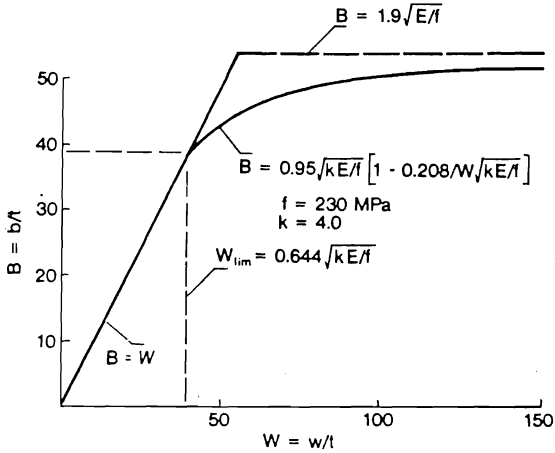

Figure 2 Effective Width Expression for Stiffened Compressive Elements

Elements stiffened on one edge by a web or flange and on the other by an edge stiffener

This type of compressive element is typically encountered with individual sections such as channel or Z-shapes, as shown in Figure 3. The approach is to use the basic effective width expression, Eq. (1), with modified plate buckling coefficients, k.

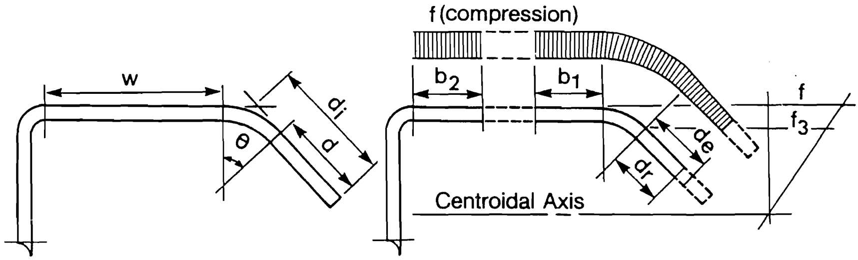

Figure 3 Example of Edge-Stiffened Flange Element Under Uniform Compressive Stress

Simple lip stiffeners and other stiffener shapes can be used as possible edge stiffeners. This section of the 1989 edition of S136 [4] contains a number of major improvements over the 1984 edition [3], as follows.

(a) The requirements for adequate stiffeners and the effective area of stiffeners have been brought together in one treatment.

(b) The requirements for adequate stiffeners have been revised to reflect recent research.

(c) The concept of the partially stiffened element has been introduced to account for the transition in behaviour between an unstiffened and a fully stiffened element.

An edge stiffener is defined as adequate if out-of-plane distortions are prevented and if the stiffened element will carry the same load as that of an identical element stiffened by a web or flange along both edges. For less than adequate edge stiffeners, the element is said to be partially stiffened and some reduction in load carrying capacity of the stiffened element results. Partially stiffened elements typically fail in a distortional mode with both the element and the stiffener buckling out-of-plane simultaneously. Three cases are presented, depending on the slenderness of the plate element being stiffened. More background information is provided in Reference [13].

The effective widths, b_1 and b_2, the reduced effective width, d_r, and the reduced effective area, A_r, must be determined in accordance with the following. See Figure 3.

Case I

When $W \leq W_{\text{lim1}}$ (no edge stiffener required)

$b_1 = b_2 = w/2$

$d_r = d_e$ for simple lip stiffener

$A_r = A_{es}$ for other stiffener shapes

An element with a low W ratio, $W \leq W_{\text{lim1}}$, is fully effective even as an unstiffened element and any stiffener is therefore adequate. Only the stiffener itself is checked for local buckling. For this case, the longitudinal stresses are uniform except for an edge stiffener with $d_i/w > 0.25$, which can destabilize the flange to which it is attached. There is no provision in the Standard [4] to account for this destabilizing effect and testing may be required.

Case II

When $W_{\text{lim1}} < W \leq W_{\text{lim2}}$

$b_1 = I_r\, Bt/2 \leq Bt/2$

$b_2 = Bt - b_1$

$d_r = d_e I_r \leq d_e$ for simple lip stiffener

$A_r = A_{es}\, I_r \leq A_{es}$ for other stiffener shapes

Where

$I_r = I_s / I_a$

$I_a = 399t^4(W/W_{\text{lim2}} - 0.33)^3$

An element with an intermediate W ratio, $W_{\text{lim1}} < W \leq W_{\text{lim2}}$, is fully effective as a stiffened element if it has an adequate stiffener, $I_r \geq 1$, such that $d_i/w \leq 0.25$. A stiffener with $d_i/w > 0.25$ can destabilize the flange to which it is attached. This destabilizing effect is accounted for by a reduction in the plate buckling coefficient, k. For the partially stiffened case, $I_r \leq 1$, both the plate buckling coefficient, k, and the effective area of the stiffener are reduced.

Case III

When $W > W_{\lim 2}$

b_1, b_2, d_r, A_r, I_r are as defined in Case II
with
$I_a = t^4[115(W/W_{\lim 2}) + 5]$

An element with a large W ratio, $W > W_{\lim 2}$, is not fully effective even with an adequate stiffener. The special considerations discussed under Case II, such as the effect of a stiffener that is too long, are also applicable.

The expressions for the moment of inertia of an adequate stiffener for both Case II and Case III were derived by Desmond, Pekoz and Winter [7,8,9].

Where for Cases I, II and III

b_1, b_2 = effective widths illustrated in Figure 3.
$W_{\lim 1} = 0.644\sqrt{kE/f}$ with $k = 0.43$
$W_{\lim 2} = 0.644\sqrt{kE/f}$ with $k = 4$
B = effective width ratio calculated in accordance with Eq. (1) with k determined as follows.

1) k for simple lip stiffeners is determined in accordance with Table 1.

Table 1. Buckling Coefficients for Simple Lip Stiffeners

		$d_i/w \le 0.25$	$0.25 < d_i/w \le 0.8$
Case II	$I_r \ge 1$	$k = 4$	$k = 5.25 - 5(d_i/w)$
	$I_r < 1$	$k = 3.57(I_r)^{1/2} + 0.43$	$k = [4.82 - 5(d_i/w)](I_r)^{1/2} + 0.43$
Case III	$I_r \ge 1$	$k = 4$	$k = 5.25 - 5(d_i/w)$
	$I_r < 1$	$k = 3.57(I_r)^{1/3} + 0.43$	$k = [4.82 - 5(d_i/w)](I_r)^{1/3} + 0.43$

Note: In Table 1, $d/t \le 14$.

The limit of $d/t \le 14$ in Table 1 is based on the work of Willis and Wallace [16].

2) k for other stiffener shapes is determined as follows:

$k = 3.57(I_r)^n + 0.43 \le 4$

Where
for Case II $n = 0.50$
for Case III $n = 0.33$

Elements with one intermediate stiffener and stiffened on each edge by a web or flange

The use of intermediate stiffeners, as well as edge stiffeners, can dramatically improve the structural efficiency of a cold formed steel member. The approach here is similar to edge stiffeners discussed in 3.2.2 above (i.e., the basic effective width expression, Eq. (1), is used with appropriately modified plate buckling coefficient, k). Again, three cases are presented, depending on the slenderness of the plate element being stiffened.

The effective width ratio, B, must be determined in accordance with the following. (See Figure 4.)

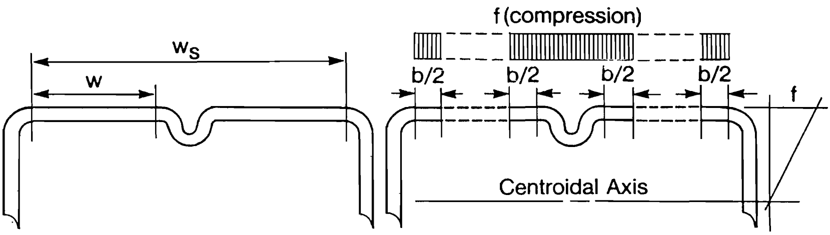

Figure 4 Example of Stiffened Flange Element with One Intermediate Stiffener Under Uniform Stress

Case I

When $W_s \leq W_{\lim}$ (no intermediate stiffener required)

$b = w$

$A_r = A_{es}$

Case II

When $W_{\lim} < W_s \leq 3W_{\lim}$

$b = Bt$

$k = 3(I_r)^{1/2} + 1 \leq 4$

$I_r = I_s/I_a$

$I_a = 50t^4[W_s/W_{\lim} - 1]$

$A_r = A_{es}I_r \leq A_{es}$

Case III

When $W_s > 3W_{\lim}$

$b = Bt$

$k = 3(I_r)^{1/3} + 1 \leq 4$

$I_r = I_s/I_a$

$I_a = t^4[128(W_s/W_{\lim}) - 285]$

$A_r = A_{es}I_r \leq A_{es}$

Where for Cases I, II and III

$W_{\lim} = 0.644\sqrt{kE/f}$ with $k = 4$

B = effective width ratio calculated in accordance with Eq. (1) with $W = w/t$ and k as calculated above.

It should be noted that, due to the lack of sufficient experimental data, the design of compressive elements with locally unstable intermediate stiffeners is not included in the 1989 edition of S136 [4].

Unstiffened Elements Under Uniform Stress

This is identical to the provisions for unstiffened elements in the 1984 edition of S136 [3] except that the plate buckling coefficient, k, has been changed from 0.5 to 0.43 to be consistent with the AISI Specification [1].

The effective width, $b = Bt$, must be determined in accordance with Eq. (1) with $k = 0.43$ and $W = w/t$. See Figure 5.

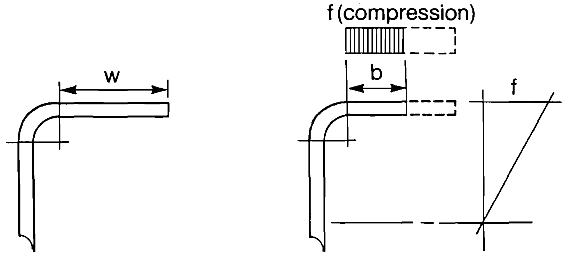

Figure 5 Example of Unstiffened Flange Element Under Uniform Stress

Elements Under Stress Gradient

The effective width approach used in this section is a fundamental departure from the 1984 edition of S136 [3] where the gross area of the web was used in conjunction with a reduced stress to account for the post-buckling strength. The adoption of the effective width approach for webs is the final step in unifying all compressive elements under the umbrella of Eq. (1). Test results reported by a number of researchers were evaluated by Cohen and Pekoz [5] for webs connected to stiffened, partially stiffened and unstiffened flanges. More background information is contained in Reference [13]. The procedure used in S136 [4] has been streamlined somewhat from that used by Cohen and Pekoz [5] and by the 1986 AISI Specification [1].

Stiffened elements

When $W > W_{\text{lim}}$, the effective widths, b_1 and b_2, must be determined in accordance with the following:

(a) For Webs (f_1 in compression and f_2 in tension - see Figure 6).

$b_1 = Bt/(3 + q)$

$b_2 = Bt/(1 + q) - b_1$

$k = 4 + 2(1 + q)^3 + 2(1 + q)$ when $0 \leq q \leq 1$

$k = 6(1 + q)^2$ when $1 < q \leq 3$

(b) For Other Stiffened Elements (f_1 and f_2 in compression - see Figure 7)

$b_1 = Bt/(3 - q)$

$b_2 = Bt - b_1$

$k = 4 + 2(1 - q)^3 + 2(1 - q)$

Where for Cases (a) and (b)

B = effective width ratio calculated in accordance with Eq. (1), with $f = f_1$ and k as calculated above.

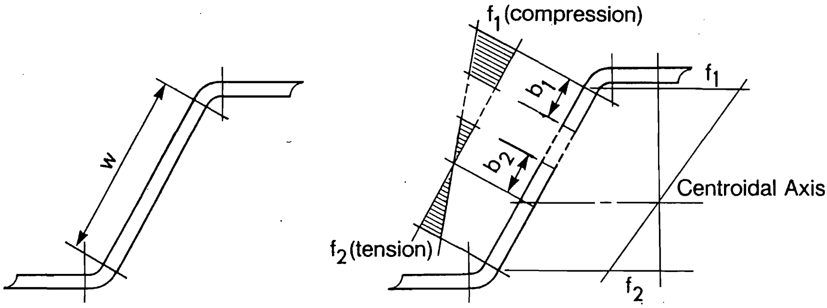

Figure 6 Example of Stiffened Web Element Under Stress Gradient

b_1,b_2 = effective widths illustrated in Figures 6 and 7.

q = $|f_2/f_1|$

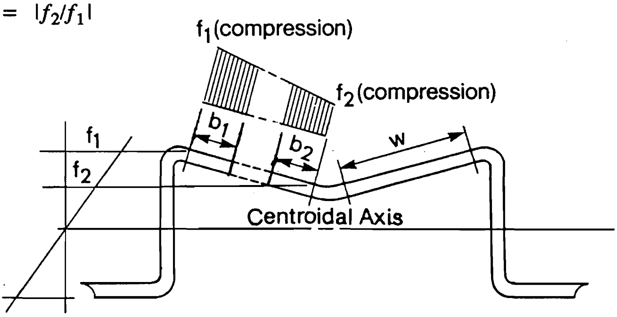

Figure 7 Example of Stiffened Flange Element Under Stress Gradient

In Figure 6, f_1 is in compression and f_2 in tension, while in Figure 7, f_1 and f_2 are both in compression, with $f_1 > f_2$. For strength determination, f_1 and f_2 are calculated using factored loads and effective section properties; for serviceability determination, f_1 and f_2 are calculated using specified loads and effective section properties.

Unstiffened Elements and Edge Stiffeners Under Stress Gradient

Due to a lack of experimental data, unstiffened elements under stress gradient are conservatively treated as uniformly compressed unstiffened elements with the stress, f, equal to the maximum stress in the element. For example, for an edge stiffener under stress gradient, see Figure 3, the effective width of the edge stiffener, $d_e = Bt$, is to be determined in accordance with Eq. (1) with $k = 0.43$ and $f = f_3$.

MEMBERS IN BENDING

In this edition of S136 [4], members in bending have been divided into two specific categories

(a) laterally supported members; and

(b) laterally unsupported members.

Laterally supported members may be designed based on the initiation of yielding or on the basis of inelastic reserve capacity, which remain the same as in the 1984 edition [3]. Considerable revisions, however, have been made in the case of laterally unsupported members.

Laterally Unsupported Members ($M_r = \phi S_c F_c$)

For symmetrical, I, Z or singly symmetric shaped single-web members, F_c, must be calculated as follows:

(a) When $F_b > F'/2$

$$F_c = F' - \frac{(F')^2}{4F_b} \leq F_y$$

(b) When $F_b \leq F'/2$

$$F_c = F_b$$

Figure 8 gives a graphic illustration of the lateral buckling behavior of I, Z and C-sections.

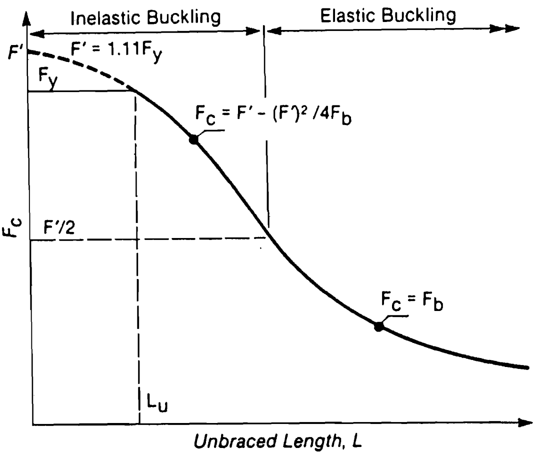

Figure 8 Lateral Buckling of I, Z and C-Sections

Bending about centroidal axis perpendicular to web

The calculation of the elastic critical lateral buckling stress, F_b, is divided into three distinct section categories as follows:

(a) For Doubly Symmetric I-Sections

$F_b = 0.833 C_b F_{be}$

$$F_{be} = \frac{\pi^2 E d I_{yc}}{L^2 S_{xc}} \; ; \qquad L_u = 0.545 \sqrt{\frac{C_b \pi^2 E d I_{yc}}{S_{xc} F_y}}$$

Where L_u is the maximum unbraced length to preclude lateral buckling.

(b) For Singly Symmetric Sections such as Channels

$$F_b = \frac{0.833}{S_{xc}} C_b r_o A \sqrt{F_{ey} F_t} \; ; \qquad L_u = \left\{ \frac{GJ}{2C_1} + \left[\frac{C_2}{C_1} + \left(\frac{GJ}{2C_1} \right)^2 \right]^{1/2} \right\}^{1/2}$$

Where

$$C_1 = \frac{11.3 F_y^2 K_y^2 S_{xc}^2}{C_b^2 A \pi^2 r_y^2 E} \; ; \qquad C_2 = \frac{\pi^2 E C_w}{K_t^2}$$

(c) For Point-Symmetric Z-Sections

$$F_b = \frac{0.833}{2S_{xc}} C_b r_o A \sqrt{F_{ey} F_t}$$

F_b of (b) above also applies to Z-sections, however, since Z-beams tend to deflect and twist laterally more easily, even between brace supports, the buckling strength will be lower. For this reason, the elastic buckling stress, F_b, of (b) above is divided by 2, resulting in a conservative approach for Z-sections in bending. As above, the maximum unsupported length, L_u, for Z-sections can be calculated using the expression of (b) above with $C_1 = 4C_1$ of (b) above. C_2 remains the same.

For both (b) and (c) above,

$$F_{ey} = \frac{\pi^2 E}{(K_y L_y / r_y)^2} ; \qquad F_t = \frac{1}{A r_o^2} \left[GJ + \frac{\pi^2 E \, C_w}{(K_t L_t)^2} \right] .$$

Bending about centroidal axis parallel to web of singly symmetric sections such as channels

F_b must be calculated as follows:

$$F_b = \frac{0.833 C_b A \, F_{ex} \, C_s}{S_{yc}} \left[j + C_s \sqrt{(j)^2 + (r_o)^2 (F_t / F_{ex})} \right\}$$

$$F_{ex} = \frac{\pi^2 E}{(K_x L_x / r_x)^2} \; ; \; F_t \text{ as above}$$

where

C_s = +1 for bending causing compression on the shear-centre side of the centroid
= -1 for bending causing tension on the shear-centre side of the centroid.

The bending coefficient, C_b, is included in the expressions for F_b to account for the effect of a non-uniform bending moment over the unbraced length. Also, the 0.833 reduction factor has been included to be consistent with the treatment of elastic buckling elsewhere in the Standard [4].

OTHER CHANGES

Wall Studs

The designer is given a choice of calculating the factored compressive resistance of a stud either based on an all steel system or based on sheathing as a bracing material. An all steel design applies to conditions during construction where the sheathing material has not yet been installed and the studs are laterally braced at certain intervals, resulting in an all steel system. It also can apply when the designer chooses to neglect the sheathing in the calculations. However, if the designer considers the sheathing to provide adequate long term structural performance, a method for such a design is presented in the Standard [4], which is the same as given in the AISI Specification [1].

Arc Spot and Seam Welds

The 1984 edition of S136 [3] covered only arc spot welds with a visible nominal diameter of 20 mm, while the 1989 edition [4] covers both arc spot and arc seam welds in various sizes. The expressions for factored shear and tensile resistance are therefore more generalized than previously, with limits corresponding to the weld sizes and material thicknesses which have been tested [14,10]. There are now two criteria for determining the resistance of an arc spot or seam weld in shear. One criterion is based on the tearing of base metal around the weld and the second is based on a shear failure of the faying surface of the weld such as could occur with thicker sheet material. There are also limits on sheet thickness and on the thickness of the underlying support member. The maximum aggregate thickness of multiple plies has been set at 2.5 mm versus the 2.0 mm permitted for single sheets. Note that in all the expressions, t is the thickness of a single sheet in the case of multiple plies. If the plies differ in thickness, the average thickness can be used.

The factored resistances, V_r and T_r are to be calculated as follows:

(a) For an Arc Spot Weld

$$V_r = \phi_u 2t(d-t)\,F_u \le \phi_c\,\frac{\pi}{4}(d_e)^2(0.75F_{xx})$$

$$T_r = \phi_u\,0.67t(d-t)F_u$$

(b) For an Arc Seam Weld

$$V_r = \phi_u\,2.10t[0.25L + 0.96(d-t)]\,F_u \le \phi_c[\frac{\pi}{4}(d_e)^2 + Ld_e]0.75F_{xx}$$

$$T_r = \phi_u\,0.70t[0.25L + 0.96(d-t)]F_u$$

CONCLUSIONS

Presented in this paper are the major technical changes contained in the new Canadian cold formed steel design Standard, CAN/CSA-S136-M89 [4]. Numerous other changes and newly introduced provisions have been made and the reader is encouraged to consult Reference [4] for more detailed information. This new Standard can truly be considered international in scope in that it is based on a unified effective width concept and Limit States Design principles, as well as SI (metric) units, with the option to use any consistent system of measurement units.

ACKNOWLEDGEMENTS

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NOTATIONS

A	fully effective cross-sectional area of section (mm^2)
A_{es}	effective cross-sectional area of stiffener (mm^2)
A_r	reduced effective cross-sectional area of stiffener (mm^2)
B	effective width ratio of an element in compression (b/t)
b	effective design width (mm)
b_1,b_2	effective widths (mm) (see Figures 3, 6 and 7)
C_b	bending coefficient = $1/\omega$, can be conservatively taken as unity, but shall not exceed 2.5 when ω is calculated as: $\omega = 0.6 + 0.4M_1/M_2$ for members bent in single curvature; and $\omega = 0.6 - 0.4M_1/M_2$ for members bent in double curvature
C_w	warping constant of torsion (mm^6)
d	overal depth of a section (mm); flat width of lip stiffener (mm); surface width (diameter) of weld, not to be taken greater than 20 mm
d_e	effective width of lip stiffener (mm) (see Figure 3); effective width of weld = $0.7d$ - $1.5\Sigma t$
d_i	overal depth of lip stiffener (mm) (see Figure 3)
d_r	reduced effective width of lip stiffener (mm) (see Figure 3)
E	Young's modulus of steel (203 000 MPa)
F_{be}	elastic buckling stress (MPa)
F_c	compressive limit stress in laterally unbraced single-web (MPa)

F_{ex}	elastic buckling stress about x-axis (MPa)
F_{ey}	elastic buckling stress about y-axis (MPa)
F_t	elastic torsional buckling stress (MPa)
F_u	tensile strength of virgin steel (MPa)
F_{xx}	tensile strength of the electrode classification (MPa)
f	calculated stress in an element (MPa)
f_1, f_2	calculated stresses (MPa) (see Figures 6 and 7)
f_3	calculated stress (MPa) (see Figure 3)
G	shear modulus of steel (78 000 MPa)
I_a	required moment of inertia for an adequate stiffener that allows the adjacent compressive element to behave as a fully stiffened element. This applies to edge and intermediate stiffeners (mm^4)
I_r	I_s/I_a
I_s	moment of inertia of fully effective cross-sectional area of stiffener about its own centroidal axis parallel to the element to be stiffened (mm^4)
I_x	moment of inertia of fully effective cross-sectional area about the major centroidal axis (mm^4)
I_y	moment of inertia of fully effective cross-sectional area about its centroidal axis parallel to the web(s) (mm^4)
I_{yc}	moment of inertia of the compressive portion of the fully effective cross-sectional area about the centroidal axis of the entire section parallel to the web(s) (mm^4)
j	$\frac{1}{2I_y}\left[\int_A x^3 dA + \int_A xy^2 dA\right] + \lvert x_o \rvert$ (mm)
J	St. Venant torsion constant (mm^4)
K_t	effective length factor for torsional buckling
k	plate buckling coefficient for compressive elements
L	unbraced length of member (mm); span of beam (mm); length of weld (mm)
L_t	length of member unsupported against twisting (mm)
L_u	maximum unbraced length to preclude lateral buckling of a member in bending (mm)
M_r	factored moment resistance (N-mm)
M_1/M_2	ratio of smaller to larger moments at opposite ends of the unbraced length in the plane of bending considered
q	absolute value of the stress ratio $\lvert f_2/f_1 \rvert$
r_o	polar radius of gyration of fully effective cross-sectional area about the shear centre (mm)

r_x, r_y	radii of gyration of the fully effective cross-sectional area about the centroidal principal axes (mm)
S_c	compressive section modulus based on the moment of inertia of the effective cross-sectional area, calculated in accordance with Eq. (1) divided by the distance from the centroidal axis to the extreme compressive fibre (mm^3)
S_{xc}	compressive section modulus of the fully effective cross-sectional area about the centroidal x-axis perpendicular to the web, I_x, divided by the distance from the centroidal axis to the extreme compressive fibre (mm^3)
S_{yc}	compressive section modulus of the fully effective cross-sectional area about the centroidal y-axis parallel to the web, I_y, divided by the distance from the centroidal axis to the extreme compressive fibre (mm^3)
T_r	factored tensile resistance (N)
t	base steel thickness (mm); thickness of sheet; one sheet thickness in the case of multiple plies (mm)
V_r	factored shear resistance (N)
W	flat width ratio (w/t)
W_{lim}	limiting flat width ratio for fully effective compressive elements
W_s	flat width ratio of a flange element stiffened by webs with one intermediate stiffener (w_s/t)
w	flat width (mm)
w_s	flat width of stiffened flange element with one intermediate stiffener (mm) (see Figure 4)
Σt	total sheet thickness to be fused to the supporting member (mm)
x_o	distance from shear centre to centroid of section (mm)
α	load factors
γ	importance factor
ϕ	resistance factor for tension, bending, and shear
ϕ_c	resistance factor for connections
ϕ_u	resistance factor for other strength limit states as determined by tensile strength of material
ψ	load combination factor